AFRL-RX-TY-TP-2010-0013

**PREPRINT**

# DESIGN LIMITS FOR PRECAST CONCRETE SANDWICH WALLS SUBJECTED TO EXTERNAL EXPLOSIONS

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**FEBRUARY 2010**



This work has been submitted for presentation at the ASCE Structures Congress, 12-14 May 2010 in Orlando, FL.

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TYNDALL AIR FORCE BASE, FL 32403-5323

| REPORT DOCUMENTATION PAGE | *Form Approved* *OMB No. 0704-0188* |
|---|---|



| 1. REPORT DATE *(DD-MM-YYYY)* | 2. REPORT TYPE | 3. DATES COVERED *(From - To)* |
|---|---|---|
| 01-FEB-2010 | Conference Paper PREPRINT | 01-JUN-2008-- 31-DEC-2009 |

| | |
|---|---|
| **4. TITLE AND SUBTITLE**<br>Design Limits for Precast Concrete Sandwich Walls Subjected to External Explosions (PREPRINT) | **5a. CONTRACT NUMBER**<br>FA4819-09-C-0032 |
| | **5b. GRANT NUMBER** |
| | **5c. PROGRAM ELEMENT NUMBER**<br>62012F |
| **6. AUTHOR(S)**<br>*Naito, Clay; *Beacraft, Mark; **Hoemann, John | **5d. PROJECT NUMBER**<br>4918 |
| | **5e. TASK NUMBER**<br>C1 |
| | **5f. WORK UNIT NUMBER**<br>Q103013 |
| **7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)**<br>*Department of Civil and Environmental Engineering, Lehigh University, Bethlehem, PA 18015; **U.S. Army Engineer Research & Development Center, CEERD-GS-V, 3909 Halls Ferry Road, Vicksburg, MS 39180 | **8. PERFORMING ORGANIZATION REPORT NUMBER** |
| **9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)**<br>Air Force Research Laboratory<br>Materials and Manufacturing Directorate<br>Airbase Technologies Division<br>139 Barnes Drive, Suite 2<br>Tyndall Air Force Base, FL 32403-5323 | **10. SPONSOR/MONITOR'S ACRONYM(S)**<br>AFRL/RXQF<br>**11. SPONSOR/MONITOR'S REPORT NUMBER(S)**<br>AFRL-RX-TY-TP-2010-0013 |

**12. DISTRIBUTION/AVAILABILITY STATEMENT**

Distribution Statement A: Approved for public release; distribution unlimited.

**13. SUPPLEMENTARY NOTES**

Ref AFRL/RXQ Public Affairs Case # 10-032. Document contains color images. For presentation at ASCE Structures Congress, 12-14 May 2010, in Orlando FL.

**14. ABSTRACT**

The use of precast/prestressed concrete and tilt-up concrete for exterior walls is common practice in the United States. This form of construction provides an economical, rapid, and high quality building technique making it ideal for military and government facilities. In most cases these building systems must be designed against a potential explosive demand. Current design recommendations are very restrictive when using precast concrete components due in large part to the lack of experimental research data. To address this issue, a series of over 50 experiments were conducted to assess the failure modes and load and deformation capacity of wall panel systems. Single span and multi-span panels were examined. Foam type, tie type, and reinforcement were varied to provide a thorough understanding of the effects of these variables on the failure modes of the panels. The response of the systems was found to be sensitive to the insulation foam used and the failure mode of the shear ties. The results indicate that insulated precast concrete panels exceed the current response limits used by the Army for Anti-Terrorism and Force Protection (ATFP) applications.

**15. SUBJECT TERMS**

design limits, PCI, sandwich panels, precast, concrete, precast panels, blast design, blast

| 16. SECURITY CLASSIFICATION OF: | | | 17. LIMITATION OF ABSTRACT | 18. NUMBER OF PAGES | 19a. NAME OF RESPONSIBLE PERSON |
|---|---|---|---|---|---|
| a. REPORT | b. ABSTRACT | c. THIS PAGE | | | Paul Sheppard |
| U | U | U | UU | 13 | 19b. TELEPHONE NUMBER *(Include area code)* |

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# Design Limits for Precast Concrete Sandwich Walls Subjected to External Explosions

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## ABSTRACT

The use of precast/prestressed concrete and tilt-up concrete for exterior walls is common practice in the United States. This form of construction provides an economical, rapid, and high quality building technique making it ideal for military and government facilities. In most cases these building systems must be designed against a potential explosive demand. Current design recommendations are very restrictive when using precast concrete components due in large part to the lack of experimental research data. To address this issue, a series of over 50 experiments were conducted to assess the failure modes and load and deformation capacity of wall panel systems. Single span and multi-span panels were examined. Foam type, tie type, and reinforcement were varied to provide a thorough understanding of the effects of these variables on the failure modes of the panels. The response of the systems was found to be sensitive to the insulation foam used and the failure mode of the shear ties. The results indicate that insulated precast concrete panels exceed the current response limits used by the Army for Anti-Terrorism and Force Protection (ATFP) applications.

## INTRODUCTION

The use of insulated precast/prestressed concrete and insulated tilt-up concrete sandwich panels for exterior walls is common practice in the United States. This form of construction provides a thermally efficient and high mass wall which enhances the energy efficiency and blast resistance of the building making it ideal for military and government facilities. Current design recommendations are very restrictive when using these forms of construction due in large part to the lack of experimental research data. To address this issue a research program has been conducted to assess the performance of conventional insulated exterior wall systems under blast loading. This report presents the static performance of the wall systems subjected to pseudo-blast pressures.

## RESEARCH SIGNIFICANCE

With the increase in international terrorism incidents over the previous decade, efforts to design military, government, and high-priority civilian buildings for blast-resistance is becoming more common than ever before. In order to maximize structure design, construction, and efficiency, any design criterion for blast must be up-to-date given the latest materials, methods of construction, and known blast sources.

Considering economic efficiency, prestressed non-load bearing sandwich wall panels, referred hereinto as panels, have been chosen as the focus in this research. These panels can be constructed off-site in a quality-controlled environment, erected into place quickly, provide some level of thermal efficiency, and provide the required blast protection for the building and contents thereof, during and after a blast.

Prescribed by the US Army Corps [2008], the current design limits for panels take an appropriate conservative approach to the problem of blast. The information in this document was developed based on historical data on the performance of reinforced concrete systems. The estimates for prestressed concrete systems are based on limited data and are consequently reduced from the limitations used for reinforced concrete systems. The performance of panels was not specifically examined in the development of the design limits. The objective of this research effort is to examine the performance of insulated sandwich wall panels and use the measured performance to present new limits.

## RESPONSE LIMITS

According to the Army Corps, the performance of a concrete element in a blast environment is assessed relative to two variables—deflection ductility, $\mu$, and end rotation, $\theta$, as noted in Equation 1 and 2. Deflection ductility is defined as the maximum deflection at midspan, $\Delta_{\text{mid}}$, resulting from a uniformly distributed load divided by the yield deflection at midspan, $\Delta_{\text{yield}}$, for the cross section.

$$\mu = \Delta_{mid} / \Delta_{yield} \qquad \text{Eqn. 1}$$

The maximum end rotation is approximated relative to the $\Delta_{mid}$ and the span length L in accordance with the following expression.

$$\theta = \tan^{-1}\left[\Delta_{mid} / (L/2)\right] \qquad \text{Eqn. 2}$$

Note that the use of end rotations as opposed to deflection as a parameter for defining performance limits allows longer span elements to deform to a greater extent. This accounts for the greater flexibility available for these components.

**Table 1. Damage Level Definitions [US Army 2008]**

| Component Damage Level | Description of Component Damage | Building Level of Protection | Limit for Reinforced Concrete Element in Flexure w/ no shear reinforcement | Limit for Prestressed Concrete Element in Flexure and reinforcement index, $\omega_p$[1] $< 0.15$ |
|---|---|---|---|---|
| Superficial Damage | Component has no visible permanent damage | High | $\mu \leq 1.0$ | $\mu \leq 1.0$ |
| Moderate Damage | Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic | Medium | $\mu > 1.0$<br>$\theta \leq 2.0°$ | $\mu > 1.0$<br>$\theta \leq 1.0°$ |
| Heavy Damage | Component has not failed, but it has significant permanent deflections causing it to be unrepairable | Low | $2.0° < \theta \leq 5.0°$ | $1.0° < \theta \leq 2.0°$ |
| Hazardous Failure | Component has failed, and debris velocities range from insignificant to very significant | Very Low | $5.0° < \theta \leq 10.0°$ | $2.0° < \theta \leq 3.0°$ |
| Blowout | Component is overwhelmed by the blast load causing debris with significant velocities | Below Anti Terrorism Standards | $\theta > 10.0°$ | $\theta > 3.0°$ |

[1]Prestressed reinforcement index, $\omega_p = A_{ps}/bd_p(f_{ps}/f'_c)$

The response limits associate physical response limits with component damage and building protection levels. Table 1 summarizes the design limits for prestressed and reinforced concrete elements subject to flexure without tension membrane action [US Army 2008]. Five damage levels are defined from Superficial to Blowout. Each damage level respectively corresponds to five levels of decreasing building protection from High to Below Standard. For instance, a flexural prestressed element with a

superficial expected damage level will be designed for deflection ductility less than 1.0.

Little to none of the data used to develop the limits for prestressed concrete elements presented in Table 1 were collected from sandwich panel tests. Therefore, to make this document more applicable towards the design of prestressed panel systems, new limits will be proposed based on the results of this study.

## NON-LOAD BEARING SANDWICH WALL PANELS

Concrete sandwich wall panels are widely used across the United States for construction of building systems. The panels consist of an interior section or wythe of insulating foam and an exterior concrete wythe as illustrated in Figure 1. The interior and exterior layers are connected to each other using shear ties. Varying the amount and type of shear ties allows the interior and exterior wythe to act as a fully composite, partially composite, or non-composite system.

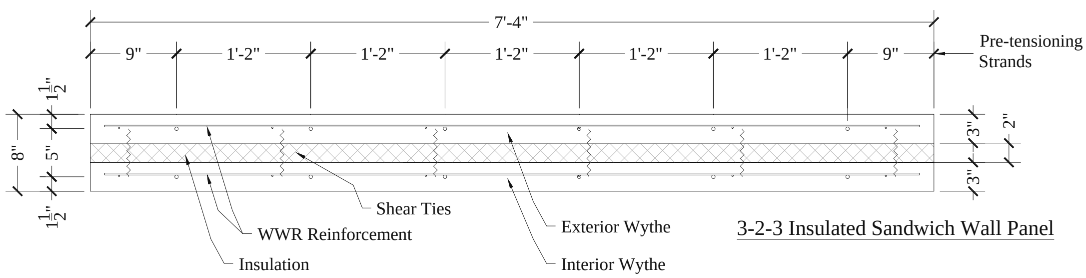



**Figure 1. Sandwich Panel (1 in = 2.54 cm)**

Concrete sandwich walls provide an ideal choice for US Government and Military facility construction. The foam sandwich provides high levels of insulation resulting in an energy efficient building envelope. The panels are prefabricated, allowing for rapid erection of the building and short construction schedules. The use of concrete provides a high inertial mass which enhances the blast resistance of the facility against external detonations. This last characteristic is essential for meeting the blast resistance requirements for Federal and Military facilities. Furthermore, while the thermal properties and construction quality has been well documented, minimal information has been generated on the performance of panels under blast demands. To quantify the blast resistance of these systems, a comprehensive experimental program is conducted on insulated sandwich wall panel systems.

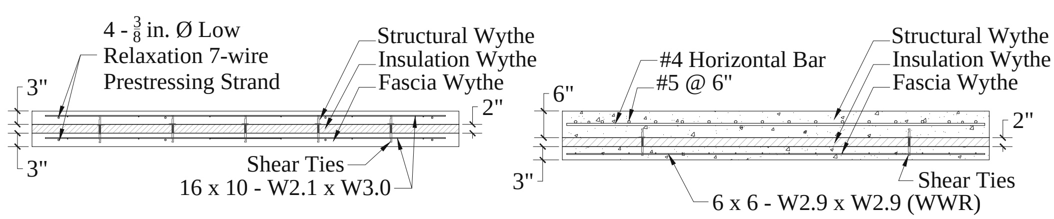
A total of 40 individual 10 ft (3.04 m) one-span panels were tested using to test 13 different panel configurations. A total of 12 individual 24 ft (7.32 m) two-span panels were tested using 5 different panel configurations. Table 2 shows the various configurations for the one-span panels. Three samples were tested for each configuration. The wythe thickness (abbreviated in inches), insulation type, amount and type of reinforcement or shear connector, or wall anchor was varied between panels. The wall panels were detailed to meet the design standards used for the Precast/Prestressed Concrete Industry (PCI) and the methods of the Tilt-Up Concrete Association (TCA). Figure 2 shows typical panel configurations for both the PCI and TCA methods of construction.

**Shear Ties**. To provide integrity between the interior and exterior concrete faces, shear ties are used. A number of different shear ties are examined in the study (Figure 3). The ties included are: (A) THERMOMASS® non-Composite Tie, (B) THERMOMASS® Composite Tie, (C) Universal Teplo Tie, (D) Dayton Superior Delta Tie, (E) Carbon Cast C-Grid®, (F) Standard Steel C-Clip. Common steel ties were used to represent commonly available and widely used shear connector. The major disadvantage of these ties is the thermal conductivity they possess; shown to increase heat transmittance through a sandwich panel by as much as 20% over glass or carbon fiber composite ties [PCI 2004]. Composite shear ties, which transmit little to no heat, were chosen to represent more thermally efficient construction. The primary disadvantage of these ties is the proprietary nature of each system.

**Table 2. Static Test Matrix for Single Span Panels**

| Specimen | Wythe Config. | Insulation | Panel Reinforcement (Longitudinal / Transverse) | Composite Ties |
|---|---|---|---|---|
| TS1 | 6-2-3 | XPS | #3 / WWR | THERMOMASS® - Non-Composite |
| TS2 | 3-2-3 | XPS | #3 / #3 | THERMOMASS® - Composite |
| PCS3a | 3-2-3 | EPS | #5 / WWR | C-Grid® |
| PCS7 | 3-3-3 | XPS | #5 / #3 | THERMOMASS® Composite |
| PCS1 | 3-2-3 | EPS | 3/8Ø strand / WWR | Steel C-clip |
| PCS2 | 3-2-3 | EPS | 3/8Ø strand / WWR | C-Grid® |
| PCS3b | 3-2-3 | EPS | 3/8Ø strand & #5 / WWR | C-Grid® |
| PCS4 | 3-3-3 | XPS | 3/8Ø strand / #3 | Steel C-clip |
| PCS5 | 3-3-3 | XPS | 3/8Ø strand / #3 | THERMOMASS® Composite |
| PCS6 | 3-3-3 | XPS | 3/8Ø strand / WWR | C-Grid® |
| PCS8 | 3-3-3 | PIMA | 3/8Ø strand / #3 | THERMOMASS® Composite |
| PCS9 | 3-3-3 | PIMA | 3/8Ø strand / WWR | C-Grid® |
| PCS10 | 3-3-3 | XPS | 3/8Ø strand / WWR | C-Grid® |
| PCS11 | 3-3-3 | XPS | 3/8Ø strand / WWR | C-Grid® |

Notes: XPS=extruded expanded polystyrene, EPS=expanded polystyrene, PIMA=polyisocyanurate, WWR=welded wire reinforcement, (1 in = 2.54 cm)




a) PCI panel b) TCA panel

**Figure 2. Typical Panel Configuration for a) PCI Panel, b) TCA Panel, (1 in = 2.54 cm)**

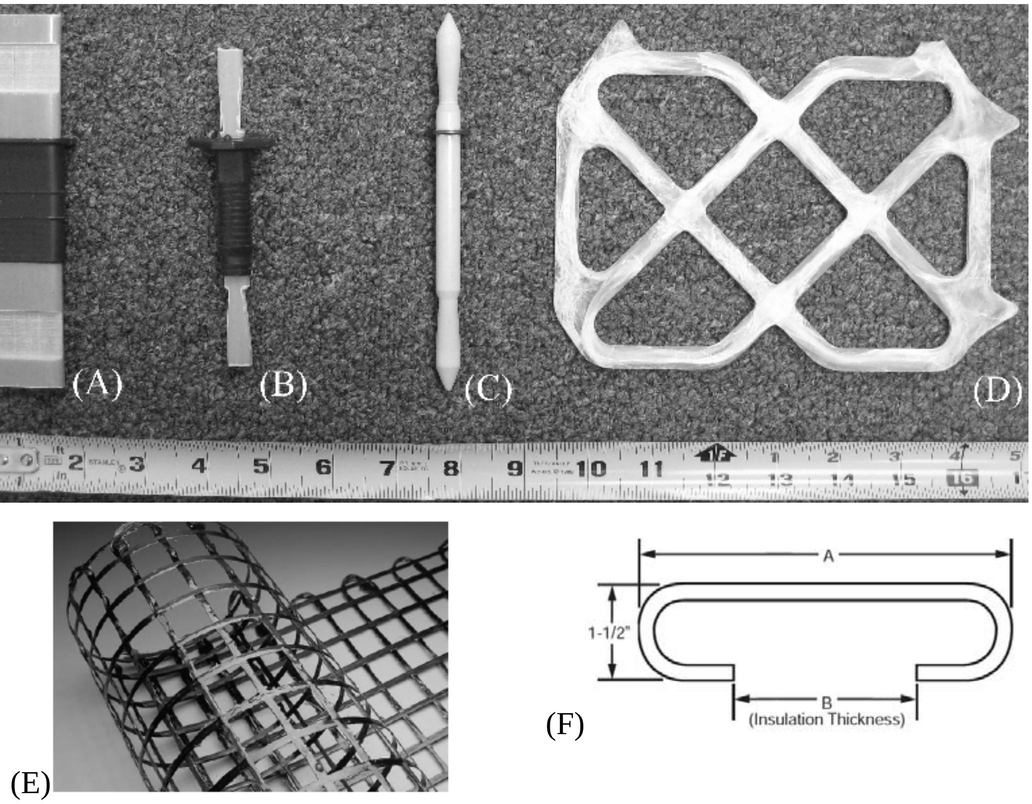



**Figure 3. Shear Tie Samples (1 in = 2.54 cm)**

## EXPERIMENT DESCRIPTION

All panels are loaded to failure using a loading tree fixture. The fixture applies equal loads at discrete points along the panel to generate an approximate uniform load. The equal point loads are applied through balanced pins which allow rotation of the individual loading arms. The result is an approximate uniform load along the span. The single span fixture is illustrated in Figure 4. The end connections on the singles span panels consist of a heavy wall steel pipe—idealized as rollers.

The two-span fixture is illustrated in Figure 5. Similar supports are used at the ends of the panel. The center connection consists of a welded detail similar to connections used in the building structure. The foundation and top floor connections were examined by fabricating the panel with the same connection on both ends. Using the same connection on each end precluded the possibility of a non-symmetric failure mode.

The measured static response of the panels includes midspan deflection, end rotation, and end slip as function of load. The static resistance measured provides a lower bound estimate of the dynamic response. At dynamic rates typical of a blast event, the concrete and steel strengths are increased by a factor of 1.1 to 1.3. The execution of static experiments is of lower cost and has inherently lower variability than dynamic studies.  Therefore, the static resistance is thoroughly examined in this study and will be validated through dynamic testing in upcoming studies.

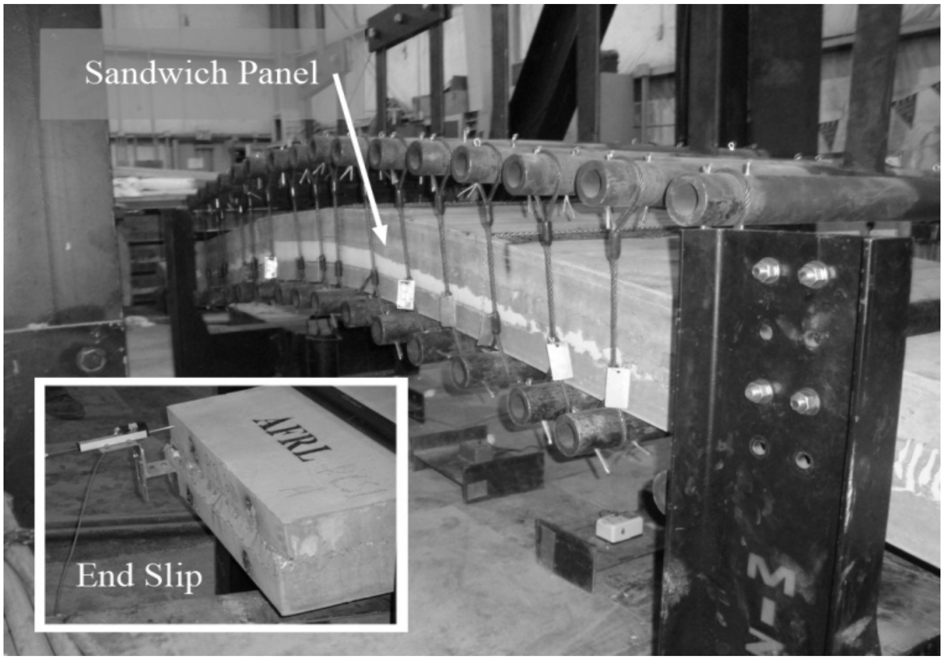



**Figure 4. One-span Loading Tree Setup**

**Figure 5. Two-span Loading Tree Setup**

## RESULTS

**Response Backbones.** In order to compare the results of the static experiments to the current response limits, the results were simplified into a backbone or multi-linear resistance function. Backbones are an approximate representation of the relationship between two variables. Whereas some methods of empirical characterization tend to focus on one variable—e.g. yield point through offset yield method—backbones attempt to capture an entire response. Figure 6 shows the backbones developed from

the measured responses for one set of single span panels. Note, the variability between PCS3 specimen is large and is not typical of the other samples.

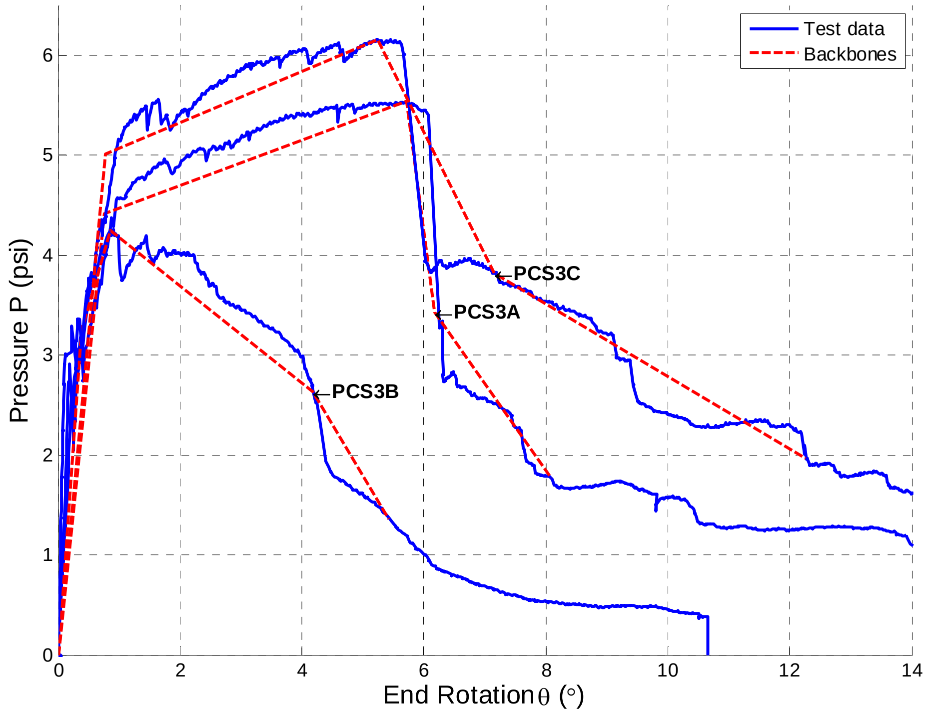



**Figure 6. Pressure Rotation Characterization (1 psi = 6.9 kPa)**

**Backbone Development.** A five point backbone of each experiment was developed and used to quantify the average response of the prestressed and reinforced concrete specimens. A procedure was developed to minimize the error between the energy of the measured performance and that of the backbone. The five points correspond to the max pressure, Pmax, secant stiffness to point K, and a two post-peak levels S and T. The points K, S, and T were chosen as a percentage of Pmax that would minimize the error between the measured pressure-rotation response and the final backbone. The error was computed as shown in equation 3.

$$error = \frac{\int P\Delta d\Delta(actual) - \int P\Delta d\Delta(backbone)}{\int P\Delta d\Delta(actual)} \qquad \text{Eqn 3}$$

Values for K, S, and T were determined as 64% Pmax, 62% Pmax, and 32% Pmax, respectively. These values provide the lowest average error on the entire dataset of one-span panel tests. A sample backbone for specimen PCS5C using these limits is illustrated in Figure 8.

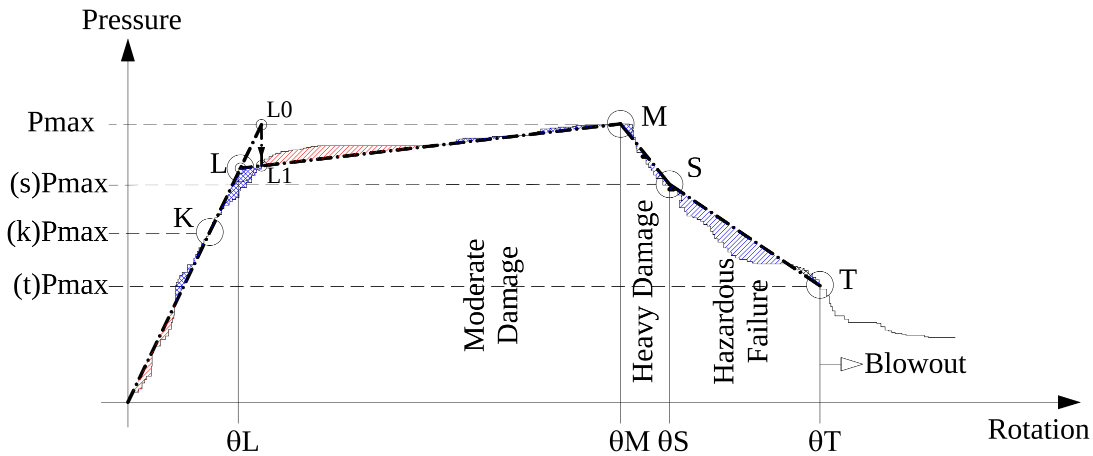



**Figure 7. Backbone Development**

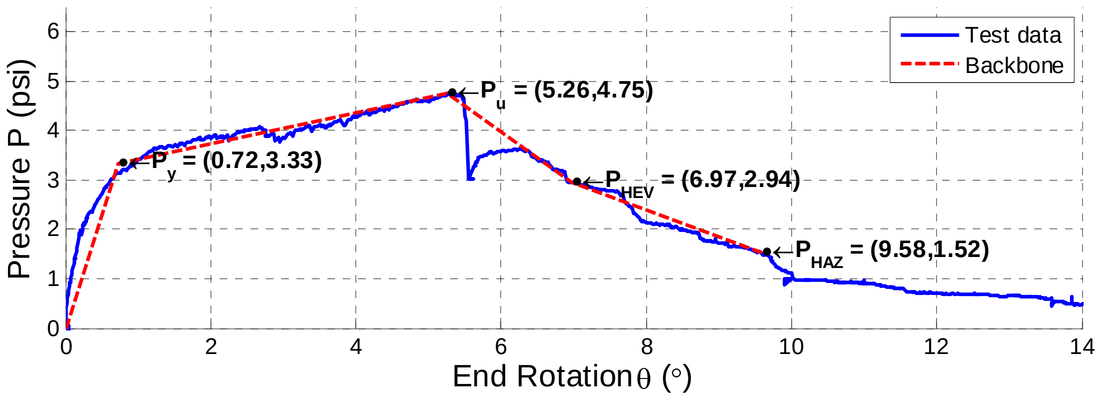



**Figure 8. Backbone with Optimum Constants (1 psi = 6.9 kPa)**

The backbone response was used to divide the panel response into different component damage levels. The division between superficial and moderate damage was not changed from the current recommendation of a ductility of 1.0. At the end of the superficial damage range, yielding initiates. Any damage prior to reinforcement yielding will be cosmetic in nature. The panels form a stable flexural response up to their ultimate capacity. Just past the ultimate strength a flexural hinge occurs near midspan. The maximum load is used as the division between moderate and heavy damage. Heavy damage corresponds to the range from Pmax to a decrease in the strength to 62% of Pmax. Hazardous damage ranges from 62%Pmax to 32% Pmax. Blowout was assumed to occur at a strength of less than 32% of Pmax.

The backbone response of each panel was averaged by type and presented in Table 3. The end rotation for each backbone level (L, M, S, and T) for both the reinforced concrete (RC) and prestressed concrete (PS) samples are summarized along with the average maximum pressure for each configuration. The results are sorted with respect to the construction type (RC or PS) and the end rotation at maximum pressure. The backbones for the reinforced and prestressed sandwich panels are shown graphically in Figure 9 and Figure 10, respectively.

| Table 3. Average Backbone Response of Single Span Panels | | | | | |
|---|---|---|---|---|---|
| Panel ID | Pmax [psi] | Backbone rotation limits for damage levels [degrees] | | | |
| | | θL | θM | θS | θT |
| PCS7 (RC) | 4.64 | 1.1 | 5.8 | 7.6 | 9.8 |
| TS1 (RC) | 2.79 | 0.4 | 5.5 | 7.8 | 12.0 |
| TS2 (RC) | 4.21 | 1.0 | 5.0 | 8.7 | 10.7 |
| PCS3a (RC) | 5.31 | 0.6 | 3.9 | 5.8 | 8.6 |
| PCS4 (PS) | 4.66 | 1.7 | 5.9 | 9.0 | 15.9 |
| PCS5 (PS) | 4.85 | 0.7 | 4.9 | 7.0 | 8.5 |
| PCS3b (PS) | 5.77 | 0.9 | 4.6 | 5.6 | 6.3 |
| PCS8 (PS) | 4.16 | 0.5 | 4.6 | 6.4 | 7.6 |
| PCS6 (PS) | 4.30 | 0.3 | 2.7 | 4.6 | 6.7 |
| PCS9 (PS) | 5.60 | 0.5 | 2.0 | 4.0 | 4.7 |
| PCS2 (PS) | 5.11 | 0.2 | 1.8 | 3.3 | 6.2 |
| PCS1 (PS) | 4.47 | 0.4 | 1.7 | 5.2 | 11.9 |
| Note: 1 psi = 6.9 kPa | | | | | |

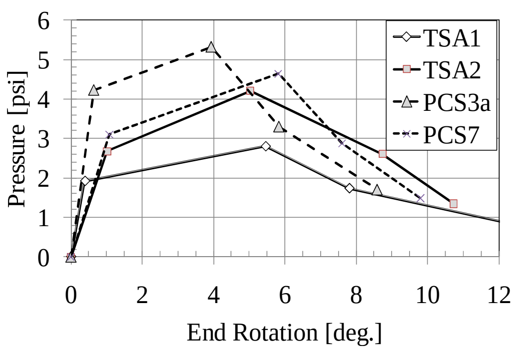



**Figure 9. Reinforced Backbones**

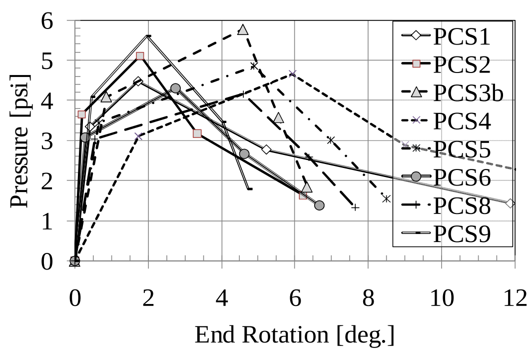



**Figure 10. Prestressed Backbones**

The 12 individual RC and 24 individual PS panel rotations were averaged to determine the general response limits of the insulated panels. The averages and standard deviations are summarized in Table 4 along with the current US Army response limitations [US Army 2008]. Recommended limits are developed for the RC and PS insulated sandwich panels based on equation 4. Due to the small sample size the proposed response limits are reduced from the average rotation levels. The proposed limits are based on a Z-Distribution with a 90% confidence level.

$$\theta \leq AVE - 1.28 \times SD \quad \textbf{Eqn. 4}$$

| Table 4. Average rotation capacity and proposed limits | | | | | |
|---|---|---|---|---|---|
| | Superficial | Moderate | Heavy | Hazardous | Blowout |
| RC Flexural Limit | μ ≤ 1.0 | θ ≤ 2.0º | 2.0º < θ ≤ 5.0º | 5.0º < θ ≤ 10.0º | θ > 10.0º |
| RC Panel Response (AVE | 0.78º ± 0.34 | 5.06º ± 1.51 | 7.50º ± 1.44 | 10.29º ± 2.12 | - |

| Table 4. Average rotation capacity and proposed limits | | | | | |
|---|---|---|---|---|---|
| | Superficial | Moderate | Heavy | Hazardous | Blowout |
| ± SD) | | | | | |
| Recommended RC Panel Limit | $\mu \leq 1.0$ | $\theta \leq 3.1^o$ | $3.1^o < \theta \leq 5.7^o$ | $5.7^o < \theta \leq 7.6$ | $\theta > 7.6$ |
| PS Flexural Limit | $\mu \leq 1.0$ | $\theta \leq 1.0^o$ | $1.0^o < \theta \leq 2.0^o$ | $2.0^o < \theta \leq 3.0^o$ | $\theta > 3.0^o$ |
| PS Panel Response (AVE ± SD) | $0.65^o \pm 0.55$ | $3.48^o \pm 1.67$ | $5.61^o \pm 1.96$ | $8.53^o \pm 3.84$ | - |
| Recommended PS Panel Limit | $\mu \leq 1.0$ | $\theta \leq 1.3^o$ | $1.3^o < \theta \leq 3.1^o$ | $3.1^o < \theta \leq 3.6^o$ | $\theta > 3.6^o$ |

## CONCLUSIONS

A series of experiments was conducted on insulated sandwich wall panels with prestressed and reinforced concrete details. The research characterized the static resistance of panels with variations in insulation, shear tie, and geometry. The results of the research show that the average measured response limits exceed the current allowable blast response limits for reinforced and prestressed concrete components under flexure. New response limits were developed based on a Z-Distribution with a 90% confidence level. The proposed response limits for insulated sandwich wall panels exceed the currently accepted limits. Two major conclusions can be drawn from this knowledge.

First, the results indicate that insulated sandwich wall panels provide adequate deformation capability to meet current blast response criteria. At a minimum, insulated sandwich wall panels can be used in accordance with the current blast design limitations. Second, the current blast response limits may potentially be increased to the proposed limits. These new limits provide information useful for improving the efficiency and economy of an already efficient and economic blast-resistant construction system. This cost-saving potential may also lead to more consideration for blast resistance being given to sandwich panels rather than other alternatives.

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US Army Corps of Engineers, "Single Degree of Freedom Structural Response Limits for Antiterrorism Design," Report PDC-TR 06-08, October, 2008.